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UPDATES TO MODELING PARAMETERS AND ACCEPTANCE CRITERIA FOR NON-DUCTILE AND SPLICE-DEFICIENT CONCRETE COLUMNS

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Abstract

The American Concrete Institute's Committee 369 is working on a new national standard that will be the source document for the concrete provisions of the ASCE/SEI 41 standard on "Evaluation and Retrofit of Existing Building". The first edition of the standard is expected to be published in 2017. As part of that effort, the nonlinear modeling parameters in ASCE/SEI 41-13 defining the lateral-load versus lateral-strength response of concrete columns were updated. The proposed new modeling parameters target median response values as opposed to the current conservative estimates, so as not to skew analytical responses. In this new approach, conservatism is introduced in the acceptance criteria associated with the modeling parameters. Independent parameters were produced for spirally reinforced circular columns, which exhibited deformation capacities roughly 30% larger than those of comparable rectangular columns. A new methodology for assessing splice deficiencies in concrete columns was introduced. The methodology accounts for loss of bond strength in regions of inelastic hinging and damage. Modeling parameters for column with splice deficiencies were modified based on the new methodology from those currently in the ASCE/SEI 41-13 standard.

Keywords: concrete, column, seismic, modeling parameters, acceptance criteria

1. Introduction

Under an agreement between the American Concrete Institute (ACI) and the American Society of Civil Engineers (ASCE), ACI committee 369 entitled "Seismic Repair and Rehabilitation" will publish a standard in 2017 that will be the source document for the concrete provisions of the ASCE/SEI 41 Standard "Seismic Evaluation and Retrofit of Existing Buildings" [1]. As part of the agreement, the ACI 369 committee used the ASCE/SEI 41-13 concrete provisions as a starting point and has been updating them for publication in 2017. This new agreement is intended to draw on the expertise of both the ASCE/SEI 41 and ACI 369 committees, to accelerate the rate of updates in the concrete provisions used nationally for seismic retrofit, and enhance the technical content. The ASCE/SEI 41-13 standard is currently the leading document used in the United States for seismic evaluation and retrofit of existing buildings. The General Services Administration (GSA) now requires that all federal buildings be evaluated for seismic vulnerability using the ASC/SEI 41 standard [2].

As part of that effort, updates to the nonlinear modeling parameters and acceptance criteria for concrete columns have been balloted and approved for publication in the 2017 versions of the ACI 369 and ASCE/SEI 41 standards. These new column criteria are presented herein.

ASCE/SEI 41-13 specifies the nonlinear modeling parameters (MP) (a_{nl}, b_{nl}, and c_{nl}) that define the lateral force versus lateral deformation relation for various members (Figure 1). For columns, the a_{nl} parameter represents the equivalent inelastic rotation at which lateral strength loss is initiated, while the b_{nl} parameter represent the equivalent inelastic rotation at which axial failure is initiated. c_{nl} defines the residual lateral strength of the member. Acceptance criteria (AC) are defined in the standard with respect to the modeling parameters and specify acceptable levels of deformation for various performance objectives (i.e., Immediate Occupancy, IO, Life Safety, LS, and Collapse Prevention, CP).

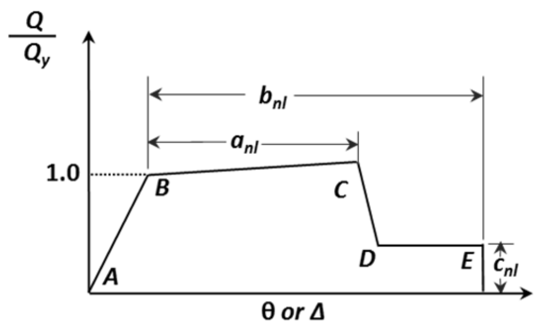

Fig. 1 - ASCE/SEI 41 backbone curve

Current nonlinear modeling parameters in ASCE/SEI 41-13 are based on conservative and inconsistent probabilities of exceedance. Consequently, when modeling the behavior of a structure, member backbone force-deformation curves are skewed from their expected shapes by varying and inconsistent amounts, which can lead to erroneous failure sequences and skewed global behavior. ACI committee 369 has elected to re-define MP values for all members to represent a median estimate wherever sufficient experimental data is available. Modeling all structural members at the median level is intended to provide a "best" estimate of the overall structural response and failure sequences. In addition to providing the median estimates on the MP, information on the spread of the error in MP estimates is provided. With this information, users can conduct parametric studies by varying MP values in cases where modes and sequences of failure can change significantly when MP are varied.

2. Columns not Controlled by Inadequate Development or Splicing

2.1 Background

Column nonlinear modeling parameters were updated in 2007 through a supplement to ASCE/SEI 41-06 [3, 4], as detailed in Elwood et al. (2007) [5]. These updates reduced the conservatism in the parameters substantially and were based on a large database of column tests [6]. However, the updated MP were still selected conservatively in the supplement. Additionally, current nonlinear modeling parameters (a_{nl} and b_{nl}) and acceptance criteria are based solely on rectangular column data.

The proposed MP equations for a_{nl} and b_{nl} were selected to provide median estimates of experimental values from an extended database of over 500 tests. Median values were selected as robust estimates of the mean. Separate provisions for circular columns were proposed as significant differences between circular and rectangular column performance was observed. Since circular columns in the database contained almost exclusively spiral reinforcement, circular-column MP equations are only recommended for use with circular columns reinforced with spirals or seismic hoops (as defined in ACI 318-14 [7]). For columns reinforced with non-seismically detailed ties, the rectangular column MP equations were recommended.

Current acceptance criteria in ASCE/SEI 41-13 are based on fixed fractions of the MP depending on the target performance objectives. For instance, AC for columns are given as 75% of the b_{nl} values for a Life Safety objective and 100% of the b_{nl} values for a Collapse Prevention objective. Because the estimates of MP exhibit different dispersions for various members, selecting a fixed fraction of those MP values for AC has resulted in varying probabilities of exceedance for the AC. Thus, proposed AC are defined through fixed probabilities of exceedance for various performance objectives and the corresponding fractions of MP values that achieve those probabilities were given.

Additional details about the derivations of MP and AC for concrete columns not controlled by inadequate development and splicing can be found in Ghannoum and Matamoros (2014) [8].

2.2 Experimental Dataset

An extensive database of column tests was used in the development of the proposed MP and AC. The database contains 319 rectangular column tests and 171 circular column tests for a total of 490 tests [9, 10] and was supplemented with 12 rectangular column tests performed to axial collapse [8, 11, 12]. None of the column tests in this database exhibited splice or anchorage deficiencies. All tests in the database were conducted quasi-statically. The database is webcast and accessible to the public ([9, 10]) and additional information about the database can be found in Sivaramakrishnan (2010) [13].

The values of a_{nl} and b_{nl} were extracted for all column tests. The a_{nl} values were taken as the equivalent inelastic column rotation at which the lateral strength degraded by 20% from peak. If a test was conducted to axial collapse, the plastic rotation at initiation of axial failure was taken as b_{nl1}. Only 36 rectangular and 9 circular columns in the database were pushed to axial failure, but the webcast database was further bolstered by 12 rectangular-column collapse tests [14, 15]. For other tests, b_{nl2} values were extracted as the inelastic rotation at the maximum lateral drift reached during testing, or alternatively the deformation at which lateral strength degraded by 75%. Additional information about the extraction of the experimental parameters can be found in Ghannoum and Matamoros (2014) [8].

2.3 Updated Nonlinear Modeling Parameters

The proposed equations for parameter a_{nl} were obtained through linear regression on the experimental values of the parameter. Continuous equations spanning the full range of column failure modes were derived to avoid stepping functions at failure boundaries, as is currently the case in ASCE/SEI 41-13. Due to the scarcity of collapse test data, the proposed relations for b_{nl} were based on the shear/friction failure model developed by Elwood and Moehle (2005) [16]. Test results from slender and 'short' columns as well as shear-critical and flexure-shear critical columns tested to axial failure have demonstrated that the proposed equations provide an adequate estimate of the drift ratio at axial failure [8, 11, 12].

For columns other than circular with spiral reinforcement or seismic hoops as defined in ACI 318-14, the resulting MP equations that will be adopted in the upcoming versions of the ACI 369 and ASCE/SEI 41 standards are:

$$a_{nl} = \left(0.042 - 0.043 \frac{N_{UD}}{A_g f_c^{'}} + 0.63\rho_t - 0.023 \frac{V_y}{V_O} \right) \geq 0.0 \tag{1}$$

V_y/V_o should not be taken less than 0.2

$$For \quad \frac{N_{UD}}{A_g f_c^{'}} \leq 0.5 \; \left\{ b_{nl} = \frac{0.5}{5 + \frac{N_{UD}}{0.8 A_g f_c^{'}} \frac{1}{\rho_t} \frac{f_c^{'}}{f_{yt}}} - 0.01 \geq a_{nl} \right. \tag{2}$$

b_{nl} in Eq. (2) should be reduced linearly for $N_{UD}/(A_g f'_c) > 0.5$ from its value at $N_{UD}/(A_g f'_c)=0.5$ to zero at $N_{UD}/(A_g f'_c)=0.7$

For reinforced concrete circular columns with spiral reinforcement or seismic hoops as defined in ACI 318-14 the equations are:

$$a_{nl} = \left(0.06 - 0.06 \frac{N_{UD}}{A_g f_c^{'}} + 1.3\rho_t - 0.037 \frac{V_y}{V_O} \right) \geq 0.0 \tag{3}$$

V_y/V_o should not be taken less than 0.2

$$For \quad \frac{N_{UD}}{A_g f_c^{'}} \leq 0.5 \; \left\{ b_{nl} = \frac{0.65}{5 + \frac{N_{UD}}{0.8 A_g f_c^{'}} \frac{1}{\rho_t} \frac{f_c^{'}}{f_{yt}}} - 0.01 \geq a_{nl} \right. \tag{4}$$

b_{nl} in Eq. (4) should be reduced linearly for $N_{UD}/(A_g f'_c) > 0.5$ from its value at $N_{UD}/(A_g f'_c)=0.5$ to zero at $N_{UD}/(A_g f'_c)=0.7$

Where V_0 is the shear strength at low deformations proposed by Sezen and Moehle (2004) [17] and given as:

$$V_0 = \alpha_{Col} \left(\frac{A_v f_{yt} d}{s} \right) + \left(\frac{6\sqrt{f_c^{'}}}{M_{UD}/V_{UD} d} \right) \sqrt{1 + \frac{N_{UG}}{6\sqrt{f_c^{'}}}} \, 0.8 A_g \, (lb/in.^2 \, units) \tag{5}$$

$M_{UD}/V_{UD}\,d$ is the largest ratio of moment to shear times the effective depth for the column under design loadings, but shall not be taken greater than 4 or less than 2

As can be seen in the equations above, the inelastic rotations at initiation of lateral strength loss (a_{nl}) are governed by the axial load ratio, the transverse reinforcement ratio, and the ratio of shear demand at column yield (V_y) to shear strength at low deformations (V_0). These influential parameters are the same as those currently used to interpolate a_{nl} values in ASCE/SEI 41-13. As such the new equation for non-spirally reinforced columns (Eq. (1)) does not produce drastically different MP than current values, but rather shifts the values to achieve a median, rather than a conservative estimate. On the other hand, the a_{nl} values obtained from the circular column Eq. (3) are about 30% larger than those from Eq. (1). This is due to the improved deformation capacities observed for spirally reinforced columns compared with other columns [8]. The fit between the proposed equations and experimental MP can be found in Ghannoum and Matamoros (2014) [8]. The dispersion for the estimates are provided in Table 1 through multipliers on the equations that achieve certain probabilities of exceedance.

Table 1 - Multipliers for concrete column modeling parameters to achieve specific probabilities of exceedance

	Modeling Parameter	Multiplier on Eq. 1 to 4 to Achieve Probability of Exceedance		
		40%	25%	10%
Columns not controlled by inadequate development or splicing along the clear height				
Reinforced concrete columns other than circular with spiral reinforcement or seismic hoops as defined in ACI 318-14	a_{nl}	0.80	0.62	0.47
	b_{nl}	0.80	0.70	0.5
Reinforced concrete circular columns with spiral reinforcement or seismic hoops as defined in ACI 318-14	a_{nl}	0.70	0.57	0.42
	b_{nl}	N.A.*	N.A.*	N.A.*
Columns controlled by inadequate splicing along the clear height				
All Columns	a_{nl}	0.62	0.5	0.33
	b_{nl}	N.A.*	N.A.*	N.A.*

* Multipliers not available due to limited test data

2.4 Updated Nonlinear Acceptance Criteria

Acceptance criteria were selected at the 10th percentile of the modeling parameter b_{nl} for the Life Safety performance objective and the 25th percentile of the Collapse Prevention performance objective. Based on values in Table 1, these corresponded to $0.5b_{nl}$ for Life Safety and $0.7b_{nl}$ for Collapse Prevention. The use of a minimum axial load ratio of 10% was recommended when calculating b_{nl} values for AC to cap the acceptable deformations at low axial loads.

3. Columns Controlled by Inadequate Splicing

3.1 Background

Current ASCE/SEI 41-13 development and splice deficiency categorization procedures, which were implemented though a supplement to ASCE/SEI 41-06 [3-5, 18], involve two steps:

1- To be considered not controlled by inadequate development or splicing, deformed bars should have an available anchorage or splice length (l_b) that exceeds the development length (l_d) specified in ACI 318-14.

2- For bars that do not meet the first criteria, the stress that can be developed in the bars is determined using:

$$f_s = 1.25\left(\frac{l_b}{l_d}\right)^{2/3} f_{yl} \le f_{yl} \quad (6)$$

Where:

f_s = Stress in reinforcement

f_{yl} = Yield strength of reinforcement

l_b = Provided straight lap splice or anchorage length

l_d = Required length of development from ACI 318-14

Members with bars not satisfying 1- above and with an expected stress in those bars that exceeds the limit given in Eq. (6), are deemed controlled by inadequate development or splicing. Anchorage deficient members are given reduced nonlinear MP representing their limited deformation capacities.

A total of 39 column tests with splices were compiled to: 1) evaluate the accuracy of current ASCE/SEI 41-13 splice deficiency classification procedures, 2) evaluate the accuracy of current nonlinear modeling parameters for columns with splice deficiencies, and 3) propose improvements to the procedures and MP. Additional details about the database can be found in Al Aawar (2015) [19].

3.2 Splice Deficiency Classification

Column tests were categorized according to three failure categories: *1 = splice failure prior to flexural yielding, 2 = splice failure after flexural yielding, and 3 = no splice failure*. Figure 2(a) plots, for each failure category, the available splice length (l_b) divided by the required splice length per ASCE/SEI 41-13 (l_d) (i.e., the development length evaluated using the detailed development length equation of ACI 318-14). A value below 1.0 of the ratio indicates that splice failure is expected per ASCE/SEI 41-13. Figure 2(b) plots the ratio of splice stress capacity evaluated using Eq. (6) to the longitudinal bar yield stress for all failure categories. A value below 1.0 indicates an expected splice failure according to Eq. (6). Table 2 lists the number of tests misclassified by the two criteria.

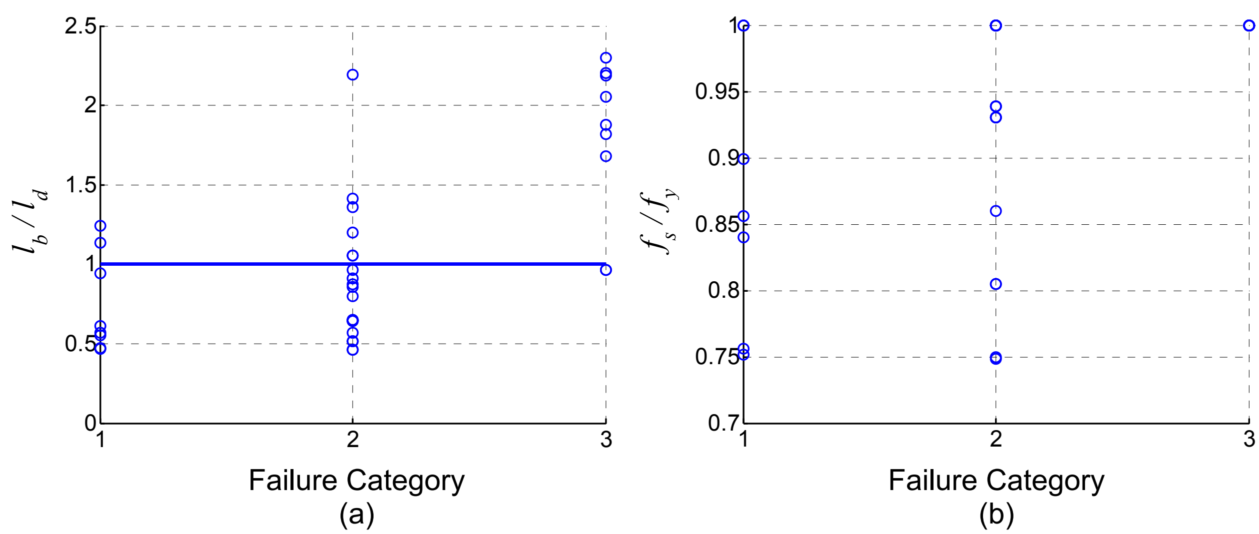

Fig. 2 - ASCE/SEI 41-13 splice classification assessments, values below 1 indicate expected splice failure

Table 2 - Classification of splice deficient columns

Number of tests with misclassified splice failure	**Category 1** (8 Total Tests)	**Category 2** (22 Total Tests)	**Category 3** (9 Total Tests)	**Sum** (39 Total Tests)
Based on l_d Criteria	2 (25%)	6 (27%)	2* (22%)	10 (26%)
Based Eq. 6 Criteria	3 (38%)	11 (50%)	0* (0%)	14 (36%)
Based on l_d Criteria and Comparing with $l_{d\text{-}deg}$	2 (25%)	1 (5%)	2* (22%)	5 (13%)
Based Eq. 7 Criteria with $l_{d\text{-}deg}$	3 (38%)	2 (9%)	2* (22%)	7 (18%)

* Tests classified as having a splice failure when experimental results did not indicate splice failure.

As can be seen in Figure 2 and Table 2, both criteria misclassified splice deficiencies, with overall errors in classification of 26% for the l_d criteria and 36% for Eq. 6. For columns deemed to be splice-deficient using Eq. 6, Figure 3 compares the experimentally derived stress in the outermost longitudinal bars at splice failure($f_s Test$), with the steel stress capacity evaluated using Eq. 6. The experimental stresses ($f_s Test$) were evaluated at the point of maximum stress in the splices (i.e., at the section of highest moment along the splice length). For all but one test in the database, the splices started at the end of the column where moments were largest and therefore, the evaluated $f_s Test$ corresponded to the moment at column end. As can be seen in the figure, Eq. 6 generates a reasonable, yet somewhat conservative, estimate of the steel stress at splice failure. Based on this finding and because the dataset contained limited tests with columns sustaining splice failure prior to flexural yielding, it was not recommend to modify the form of Eq. 6.

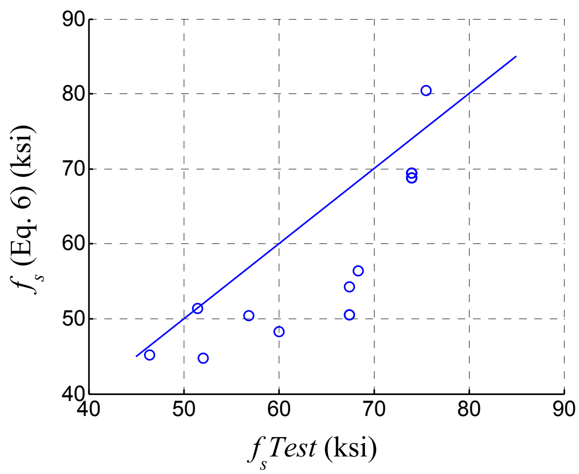

Fig. 3 - f_s using Eq. 6 versus experimental *fs* at point of maximum moment at splice failure for test classified as splice deficient according Eq. 6

The largest errors in classification were found for Category 2 tests (Figure 2 and Table 2), which sustained flexural yielding prior to splice failure. Damage in the plastic hinge region during inelastic deformations tends to weaken the bond mechanisms within the hinge region and reduce the effective splice or anchorage length. In a recent study by Sokoli and Ghannoum (2016) [20], the length of the hinge region in which bond resistance was compromised was found to be about 2/3d. This damage length is comparable to the one recommended by Ichinose (1995) [21]. The current provisions in ASCE/SEI 41-13 do not reduce the effective splice length in columns sustaining flexural hinging (i.e., Categories 2 and 3), which may explain the relatively large number of Category 2 columns misclassified as not being splice deficient.

An adjusted available lap splice length ($l_{b\text{-}deg}$) was defined for splices within plastic hinge regions (Categories 2 and 3). The degraded splice length was evaluated for splices in plastic hinge regions by subtracting from l_b a distance of 2/3d from the point of maximum flexural demand. Figure 4 plots for each failure category the available splice length, either degraded or not (l_b or $l_{b\text{-}deg}$) depending on whether flexural hinging occurred within the splice length, divided by the required splice length per ASCE/SEI 41-13 (l_d). An adjusted bar stress capacity was also defined for splices within hinge regions as:

$$f_{s-deg} = 1.25\left(\frac{l_{b-deg}}{l_d}\right)^{2/3} f_{yl} \leq f_{yl} \tag{7}$$

As can be seen in Figure 4 and Table 2, accounting for the loss of bond capacity within the hinge region resulted in significant improvements in the classification of splice deficiency for columns in Category 2.

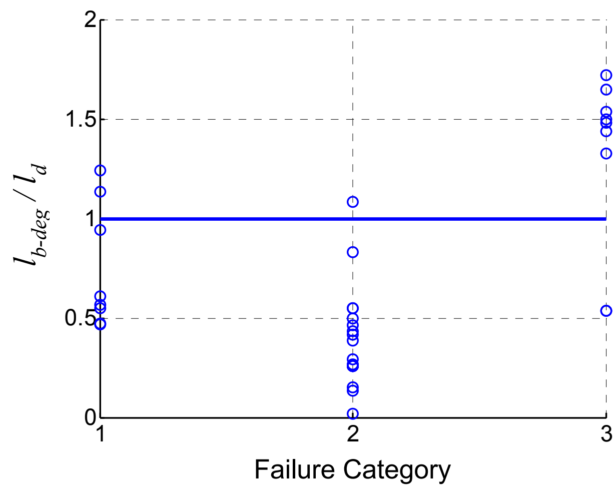

Fig. 4 - ASCE/SEI 41-13 splice classification assessments with degraded available splice length; values below 1 indicate expected splice failure

It was therefore recommended for the upcoming editions of the ACI 369 and ASCE/SEI 41 standards to classify column development and splice deficiency using the same process currently in ASCE/SEI 41-13 but accounting for the degraded anchorage length that occurs in plastic hinge regions.

3.3 Nonlinear Modeling Parameters

3.3.1 Relation for the modeling parameter a_{nl}

ASCE/SEI 41-13 relates the modeling parameter a_{nl} in columns controlled by inadequate anchorage and splicing to the axial load ratio and the transverse reinforcement ratio; with the modeling parameter a_{nl} defined as the plastic rotation at initiation of lateral strength loss. For the collected database, the parameter a_{nl} was extracted as the equivalent inelastic rotation at which lateral strength drops to 80% of the peak load (δa), with a_{nl} = (δa-δi)/(column length). The displacement δi was taken at the intersection of a secant stiffness line passing through 0.7 of the peak lateral load and the peak load. In this way, the extracted parameter a_{nl} does not reflect plastic rotations for columns that exhibit splice failures prior to flexural yielding, but rather equivalent inelastic rotations beyond peak load.

The experimental a_{nl} values versus the axial load ratio and the transverse reinforcement ratio (ρ_t) are plotted in Figure 5. As can be seen in the figure, the axial load ratio is not correlated with a_{nl}. However, the figure indicates correlation between the transverse reinforcement ratio and a_{nl}. Trends were also explored for other variables such as s/d, l_b/l_d, shear stress, and the ACI 318-14 development length confinement term. In the end, the transverse to longitudinal reinforcement ratio ($\rho_t f_{yt}$) / ($\rho_l f_{yl}$) showed the best correlation with the parameter a_{nl} and was used to generate an equation for it (Figure 6). This ratio provides a measure of the relative amount of transverse reinforcement to the amount of longitudinal bars being spliced. As can be seen in Figure 6, there is significant scatter in the data, with Category 2 columns exhibiting larger a_{nl} values for a given reinforcement ratio than Category 1 columns. A single equation for both categories of columns was however selected for simplicity. The selected relation is illustrated in Figure 6 and given below:

$$a_{nl} = \frac{1}{8}\frac{\rho_t f_{yt}}{\rho_l f_{yl}} \leq 0.025 \tag{8}$$

The dispersion for the estimates of a_{nl} using Eq. (8) are provided in Table 1 through multipliers on the equation that achieve certain probabilities of exceedance.

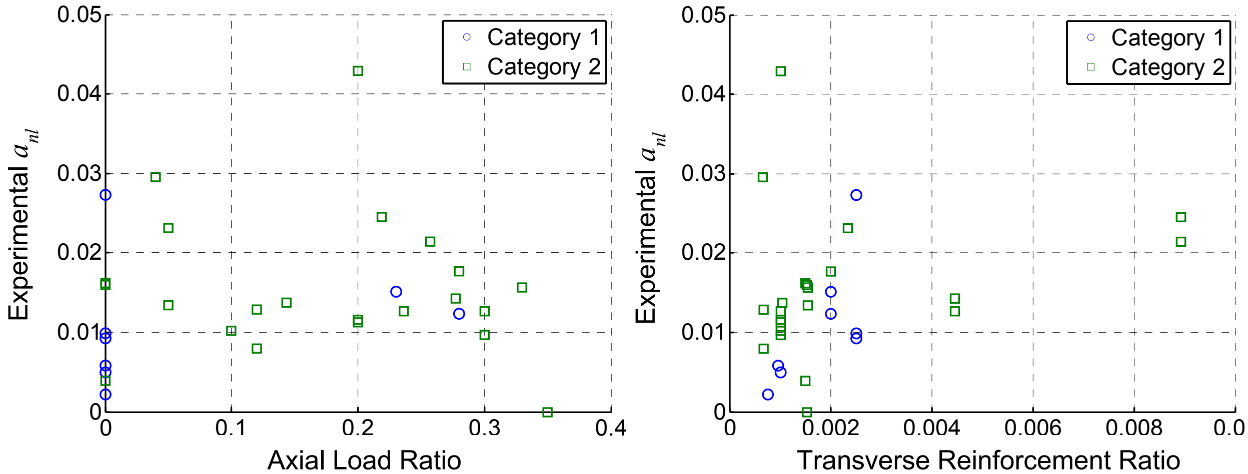

Fig. 5 - Experimental a_{nl} parameter versus the axial load ratio and the transverse reinforcement ratio (ρ_t)

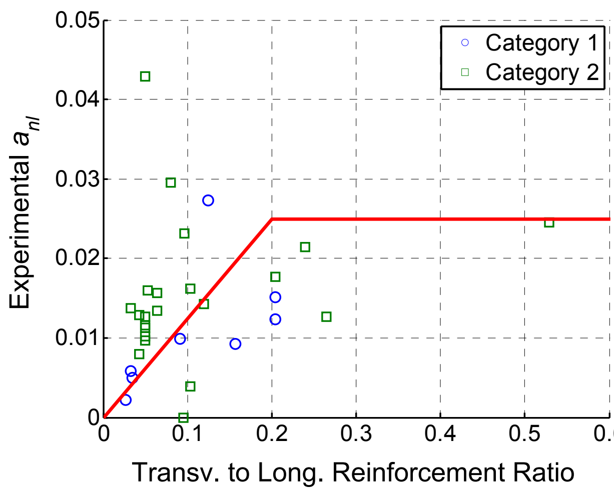

Fig. 6 - Transverse to longitudinal reinforcement ratio versus experimental values of a_{nl}

3.3.1 Relation for the modeling parameter b_{nl}

The parameter b_{nl} in ASCE/SEI 41-13 is intended to provide the plastic rotation at initiation of loss of vertical load carrying capacity. All but three tests in the database were not conducted to axial collapse. The three tests that were, are the ones conducted by Lynn (2001) [22]. For other columns in the dataset, the b_{nl} values were taken as the equivalent inelastic rotation at the end of the test or when the lateral load dropped to below 25% of peak. Therefore, most of the experimental b_{nl} values presented here can be considered a lower-bound on the actual b_{nl} values.

The current ASCE/SEI 41-13 tabularized b_{nl} values can be converted to the following equation:

$$b_{nl\ ASCE41} = -0.104\frac{P}{A_g f_c'} + 9.8\rho_t + 0.0115\Big\}{}^{\geq 0.0}_{\leq 0.06} \tag{9}$$

Figure 7(a) compares the experimentally derived b_{nl} values and the values obtained using the current ASCE 41 provisions (equation above). As can be seen in the figure, the current provisions provide overly conservative estimates of b_{nl} for the three tests conducted to collapse, as well as the majority of other tests for which lower bound b_{nl} values are only available. The relation below was proposed for b_{nl} to reduce the conservatism of the current provisions:

$$b_{nl} = -0.075\frac{P}{A_g f_c'} + 12\rho_t + 0.012 \Big\}_{\leq 0.06}^{\geq 0.0 \ \& \ a_{nl}} \tag{10}$$

Figure 7(b) compares the b_{nl} values derived using the proposed Eq. (10) with the experimental b_{nl}. As can be seen in the figure, the proposed relation reduces the conservatism considerably. One could consider a relation that increases b_{nl} values even further, but given the limited data, it was not deemed prudent to do so.

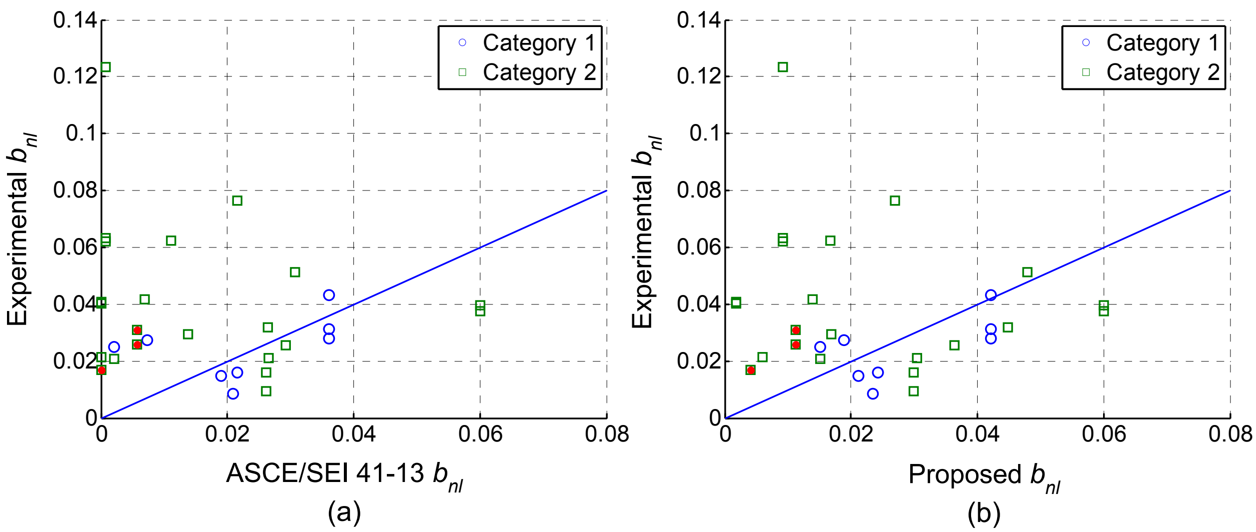

Figure 7: (a) Experimental b_{nl} values versus b_{nl} values from current ASCE/SEI 41-13 provisions. (b) Experimental b_{nl} values versus values from Eq. (10). The three tests that were conducted to axial collapse are highlights with filled markers.

4. Conclusions

The nonlinear modeling parameters (MP) and acceptance criteria (AC) for reinforced concrete columns of the ASCE/SEI 41-13 standard have been updated for inclusion into the 2017 versions of the ACI 369 and ASCE/SEI 41 standards. Updates included delivering modeling parameters through continuous equations that cover the full range of column behavior, providing median estimate values for MP, accounting for the beneficial effects on deformation capacity of spiral and circular hoop reinforcement, and selecting AC based on target probabilities of exceedance rather than as fixed fractions of MP. A dataset of over 500 column tests was used to develop the new MP and AC for column not controlled by inadequate development or splicing with their length, while a dataset of 39 column tests was used to develop the MP and AC for columns with deficient splices. Criteria for identifying deficient splices were updated to account for the loss in bond and anchorage capacity with the plastic hinge regions of columns. For columns with splice deficiencies, the MP representing the deformation at initiation of lateral strength loss was found to correlate with the transverse to longitudinal reinforcement ratio. As the ratio of transverse reinforcement to longitudinal reinforcement being spliced increased, so did the observed deformations at loss of lateral strength.

The updated MP now provide median estimates of structural response, where sufficient data was available, removing conservatism in the modeling process. Dispersion measures in the MP estimates are provided to facilitate probabilistic seismic vulnerability assessments. Conservatism is introduced consistently through acceptance criteria that are selected to achieve target probabilities of exceedance.

5. Notation

A_v	=	Area of transverse reinforcement in direction considered
A_g	=	Gross area of column, in.^{2}
d	=	Distance from extreme compression fiber to centroid of tension reinforcement. d can be taken as $0.8h$, where h is the dimension of the column in the direction of shear
f'_c	=	Compressive strength of concrete (in lb/in.^{2})
f_{yt}	=	Yield stress of transverse reinforcing reinforcement
M_{UD}	=	Member design moment accounting for the effects of lateral forces
N_{UD}	=	Maximum compressive axial load accounting for the effects of lateral forces
N_{UG}	=	Axial compression force due to gravity loads (set to zero for tension force)
s	=	Center-to-center spacing of transverse reinforcement
V_{UD}	=	Member design shear force accounting for the effects of lateral forces
V_y	=	Shear demand resulting in flexural yielding of the plastic hinges; evaluated using the longitudinal steel yield stress
α_{Col}	=	Dimensionless parameter for evaluating the effectiveness of transverse reinforcement in resisting shear forces; α_{Col} = 1.0 for $s/d \leq 0.75$, 0.0 for $s/d \geq 1.0$, and varies linearly for s/d between 0.75 and 1.0
ρ_l	=	Longitudinal reinforcement ratio = area of longitudinal steel divided by A_g
ρ_t	=	Transverse reinforcement ratio = A_v / (bd). ρ_t should not be taken greater than 0.0175 in any case nor greater than 0.0075 when ties are not adequately anchored in the core. The Equations are not valid for columns with ρ_t smaller than 0.0005

6. Acknowledgments

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